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INVESTIGATION OF FLOORBEAM HANGERS IN RAILROAD TRUSSES

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INVESTIGATION OF FLOORBEAM HANGERS IN RAILROAD TRUSSES

C. H. Sandberg,* M. ASCE

SYNOPSIS

This paper presents a review of the investigations carried on during the last several years into the causes and possible remedies for failures in floorbeam hangers of railroad truss bridges.

INTRODUCTION

Back in 1945-46 numerous failures were being reported in floorbeam hangers of railroad truss bridges. Consequently a study of floorbeam hanger failures was recommended by Com. 15, Iron and Steel Structures, of the Am. Ry. Engr. Ass'n., and arrangements made with Purdue University whereby Prof. L. T. Wyly and his staff would conduct field and laboratory tests, including theoretical analysis. The research staff of the A.A.R. assisted with some of the field tests.

Work Program

The work program of the project consisted of the following:

1) A fact finding survey among Class 1 railroads was made to secure the actual number of failures reported, location, type, etc. These failures were then analyzed.(1)

2) Strain measurements were made on floorbeam hangers of various selected railroad trusses under both static and dynamic loads.(2)(3)(4)

3) Theoretical analyses were made and correlated to actual measured stresses.(2)(3)(5)

4) Laboratory investigations and static and fatigue tests were made on models of floorbeam hangers and components to explain the types of failures occurring.(1)(5)(6)(7)(8)

The actual field and laboratory work on this program got under way in 1947 and there remains now only the writing up of a couple of remaining field tests. The A.R.E.A. has contributed to this project about $60,000.00.

Survey and Analysis of Failures Reported

In 1947 a questionnaire was sent out to the various Class 1 railroads and there were reported 171 failures in 98 spans of 50 bridges. Since that time numerous other failures have been reported. These failures occurred in the hangers of both riveted and pin-connected trusses.(1)

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Of the reported failures in the riveted truss spans only 12 of the hangers failed at the point of greatest computed bending stress, which is a short distance above the floorbeam. In the case of 10 of these it was considered that rigid frame bending was a major factor in the failures since the bending plus axial stress at the failure section was 55 ksi if computed on an elastic basis. The remaining two hangers of this group are believed to have failed due to resonant vibration. There were 79 failures in the net section through the line of the lowest rivets connecting the members to the top chord gussets. These 79 failures occurred apparently as a result of a certain number of repetition of load producing a calculated unit stress on the net section at point of failure ranging from about 9 to 24 ksi, and the greatest number occurred after being stressed to about 18 to 22 ksi. All the failures occurred after relatively few repetition of stress. In most cases the estimated number of loadings under heavy engines were 30,000 to 40,000. The dead load stress in these members was about 2 ksi so that the range of stress due to the addition of the live load was relatively much larger than is the case with most truss members. The riveted truss spans in which failures occurred were fabricated between 1899 and 1928.

The analysis of the 80 failures in 28 different pin-connected truss bridges show that these failures can be classified in the following location categories:

Fifty-five hangers failed at re-entrant cuts where the channel angles were coped to clear other members.

Ten failures were in the pin plates through the pin hole.

Eight failures were in the main material through the lowest line of rivets connecting hanger to the pin plates.

Five failures were through the end rivets in the stay plates.

Two failures were through the pin plates at the top of the hanger material.

All of the failures were in pin-connected truss spans fabricated and erected between 1880 and 1905.

In the hangers of the pin-connected truss spans the calculated maximum axial unit stress on the net section was considerably lower than the usual design stress for bridge members, and failure undoubtedly resulted from a large number of cycles of these low stresses at sections having large stress raisers such as that produced by re-entrant angle at the copes of the flanges.

The following stresses in hangers of pin-connected spans are of interest:

There were 17 failures where the calculated stress in the hanger was below 10,000 psi.

Twenty-five hangers failed where the calculated stress was between 10,000 and 15,000 psi.

There were 18 failures where the calculated tensile stress in the hanger was over 15,000 psi.

Probable Causes of the Failures in Hangers of Riveted Trusses

The failure section was always along a line through the lowest row of rivets connecting the member to the upper gusset. Failure always started at the side of a hole. After surveying and analyzing the failures, Professor Wyly and his staff evolved a hypothesis to explain these failures and to provide a remedy. The essential features were:[(1)(7)(8)]

1) Very high local stress and strain concentrations strain the metal into the plastic range and flow occurs.

2) Repeated cycles of loading and unloading restore elasticity, while retaining the stress concentrations. The latter, particularly when accompanied

by very steep stress gradients, produce high local shear stresses which when repeated many times may result in the formation of a crack.

3) The location and trajection of this crack will coincide with the path of the maximum principal tensile stress.

4) Elimination of the high local stress and strain concentrations is necessary to prevent fatigue failures from occurring.

5) A state of stress producing compression at the sides of the holes will be beneficial—it will prevent cracks from opening up.

As applied to the failures through the rivet holes the above results in the following conclusions:

It is probable that the principal source of the high stress or strain concentrations which result in the low fatigue strength of these hangers, is the bearing of the rivets at the top of the rivet holes at the failure section, probably occurring in most cases where the rivets have lost or never had their proper clamping force. While it is recognized that hot-driven rivets probably do not fill the holes in most cases, it seems likely that the rivets at the lower edge of the gusset actually bear on the material in many cases. This would be particularly true when the clamping force is low and a slip occurs between the hanger material and the gusset.

Laboratory tests that were made confirmed the fact that the strains at the sides of the holes attributable to rivet bearing were so large that they will dwarf all other causes when they occur.

Failures do not occur more often in the hangers at the top of the floorbeam in through trusses due to the reinforcing effect of the gusset or fill which almost always extends up the hanger above the floorbeam or floorbeam bracket.

Probable Causes of the Failures in Hangers of Pin-connected Trusses

The explanation for most of the failures in these hangers at low unit stresses lies undoubtedly in the use of forked ends with the two prongs bearing on the pin. Any imperfection of bearing of either prong on the pin will set up a severe local bending or racking in the member where the forked ends join the stiffly braced section of the member. In many cases bending of the forked end may be considerable due to local eccentricities. Such eccentricities arise from the addition of splice plates, coping of flanges and uneven wear which shift the center of bearing on the pin.

The effect of the addition to a forked end condition of such a severe stress raiser as a cope results in the superimposing of high local stress concentrations upon the bending stress in the forked end. In only one or two cases were copes present in members without the failure crack passing through the reentrant angle of the cope.

The axial unit stresses producing failures in the hangers of pin-connected spans are about 4 ksi to 6 ksi lower than the axial unit stresses which produced failure in riveted spans. This lower fatigue strength may be directly attributed to the presence of the large stress raisers in the pin-connected hangers. It was also especially notable that the more severe the stress raiser the lower the fatigue strength.

It appears probable that many of the floorbeam hangers of pin-connected trusses now in service having stress raisers similar to those mentioned will fail as soon as the number of repetitions of stresses becomes large enough. The failures so far reported fall into certain definite location categories, all of which are characterized by high local stress concentrations. The location

of these failures can be used as a guide for field inspection of hangers now in service.

Strain Measurements on Bridges in Service

Strain measurements were made on the floorbeam hangers of the following five truss bridges which had different make-up of hanger section:

The I.C.R.R. bridge near Galena, Ill. This is a single track 127 foot through Pratt truss bridge. The hanger section is composed of two channels tied together with occasional 10 inch deep tie plates.

The M-K-T R.R. bridge near Erie, Kansas. This is a single track 200 foot through Warren truss bridge. The hanger section is a wide flange beam.

The T. & N. O. R.R. bridge near Calumet, La. This is a single track 400 foot through Warren truss bridge. The hanger section is made up of 4 angles and a continuous web plate.

The M-K-T R.R. bridge near Dennison, Texas. This is a single track 124'-5-1/2" through Pratt truss bridge. The hanger section is composed of a wide flange beam with a cover plate on the inside flange.

The Santa Fe Ry. span near Ponca City, Okla. This is a single track 124 foot pin-connected through Pratt truss bridge. The hanger section is composed of two channels laced together.

The purpose of the tests on the floorbeam hangers of the 127' single track through Pratt truss bridge near Galena, Ill., was to investigate the effects of the form of hanger section and of the floor beam and stringer deflection upon the stress distribution in the hanger. As mentioned previously the hanger section consisted of two channels connected together only by occasional 10" tie plates. There was a solid web between the channels at the floorbeam connection.

SR 4 gages were placed at eleven sections of each hanger, a total of 228 gages being used for the static tests and 64 gages being used on the dynamic tests.

The principal conclusions from the tests at this bridge were:

This type of hanger composed of two channels or segments having no connection with each other except occasional tie plates, does not act as a unit but as two individual members which are subject to severe racking stress.

This racking is due fundamentally to the lack of adequate shear bracing for the hanger section and occurs as load is transferred from the inside channel to the outside channel.

Even though there was a substantial diaphragm between the two channels opposite the floorbeam connection, it was not sufficient to transfer half the floorbeam reaction to the outside channel. At the top of the floorbeam bracket the inside channel carried about 50% more direct load than the outside channel.

The use of a tie plated hanger section results in severe local bending in the channels at the tie plates due to the racking set up in transferring load from the inside to the outside channel.

The bending stresses resulting from rigid frame action and racking action were more than 100% of the average axial stresses in the hanger tested.

All the measured stresses can be accounted for in a rational manner but not by the usual methods of analysis.

The dynamic stresses showed the same type of stress distribution as the static stresses.

It would be expected that this type of floorbeam hanger would have poor fatigue strength due to the excessive local bending.

A rational design of the floorbeam hanger frame using this type of hanger section does not appear to be practicable.

The field measurements made on the M-K-T bridge at Erie included readings on 960 SR4 gages under static loading and on a much smaller number of gages under dynamic and vibration loading. A thorough check of both rigid frame bending and of resonant vibration of the hanger was made.

The field measurements on the T & N O R.R. bridge were on two different hangers.

The hanger in a pin-connected truss that was investigated was on a 124 foot 2 in. through Pratt truss of the Santa Fe Ry., near Ponca City, Okla.[3] Only static tests were made at this bridge.

The tested hanger was composed of two channels tied together with lacing throughout most of their length. There was a deep diaphragm between the channels at the floorbeam connection. A forked end was provided for the pin connection at U1 where the channel flanges were coped for clearance with the top chord.

The objectives and conclusions of this test can be summarized as follows:

1) Readings to determine the effectiveness of lacing in tieing two channels together indicate that the lacing bars are sufficient to make component hanger parts act as a unit.

For all practical purposes such lacing will preserve a plane section in bending.

2) The bending stresses in the hanger indiced by rigid-frame action between floorbeam and hangers run as high as 64 percent of the average $\frac{P}{A}$ value for the member.

3) The effect of stringer deflection was not found to be significant on hanger bending in the plane of the truss.

No appreciable hanger bending was observed in this plane.

4) Measured hanger stresses can be accounted for in a rational manner. Such accounting requires a modified application of rigid-frame theory to the crossframe composed of floorbeam and hangers. Modifications include the effects of:

a) Shear deformation in the hanger itself.
b) Shear deformation in the diaphragm at the floorbeam connection.
c) Imperfect fit of the pin at U_1.

5) Concentration of stress in the corners of hanger channel copes is very real. Stresses measured at these corners in the tested member were as much as 5.7 times the measured stress on a section located 4 in. above the cope. This concentration of stress greatly reduces the fatigue strength of the member.

6) By replacing the sharp corner of a cope with a smooth, large curve such stress concentration can be relieved. After modifying the copes of the tested member in this manner, stresses were reduced to 2.9 times stress measured at the section 4 inches above the cope. This method of reducing stress concentrations is applicable to existing structures and should appreciably increase the fatigue strength of the hanger members.

As so many hanger failures in pin-connected spans have occurred at reentrant cuts where the channels or angles are coped to clear other members, it may be of interest to give a little more detail of the stress measurements at these copes.

Of the 54 failures reported through the copes of hangers, analysis showed

that the unit stress varied from 6.4 ksi to 15.8 ksi with the majority of the cases at nearly the average of these two values. The preponderance of failures at coped flanges at relatively low average unit stresses emphasizes again the detrimental effect of such stress raisers on the fatigue strength of a member. The superimposing of high local stress concentrations upon the normal stresses due to direct load and bending at such points cause a stress range in the member which greatly reduces its life due to the local fatigue of the metal.

The existing cope was filed smooth to about 3/32 inch radius. SR 4 electrical resistance strain gages were used in measuring all stresses. Gages with 1/16 inch gage length were used in the cope while gages with 1/4 inch gage length were placed on each side of the flange about 3/16 inch from the corner of the cope. Gages were also placed 1/2 inch and 4 inches above the cope. The stresses recorded in the cope varied from 23.9 ksi to 32.9 ksi. The stress about 4 inches above the cope was 4.9 ksi. After these tests the cope was torch-cut to a radius of 3 to 4 inches. This cut was then ground and filed down so the surface was smooth and free from any crack or irregularities. New strain gages were placed as before and the stresses recorded in the cope were 15.7 ksi and 16.5 ksi. The stress 4 inches above the cope was 5.4 ksi.

With the original copes the stress concentration factor was about 5.7. After the copes had been cut to a larger radius the factor was reduced to about 2.9. Of more importance than the reduction in stress concentration factor is the reduction in the actual stress in the corner of the cope. The maximum stress reduced from 32.9 ksi to 16.5 ksi is about a 50% reduction in the extreme measured stress. It may well be that in many older bridges the re-entrant cope angles are sharper than the 3/32 inch radius applying in these tests. Also the readings secured were for static load and any impact would increase the values given here. Both of these factors would increase the stress concentrations above those increased here.

The method used at this bridge to secure stress relief at the copes is simple to do and could easily be applied to existing structures. For success in such a procedure it is necessary that the cut be of a large radius and the final surface smooth and free of irregularities. If a torch is used for cutting the burned surface should be ground down at least 1/4 inch below the burned metal.

It is felt that this method of stress relief could be applied to many existing structures and increase the life of the coped members, as it is reasonable to expect a member with a stress range from 0 to 16.5 ksi will withstand appreciably more cycles than one having a stress cycle of 0 to 32.9 ksi.

Theoretical Analysis

As a part of this program of investigation of floorbeam hangers it was necessary to develop a satisfactory method of analysis. Measured static stresses in the field exceeded computed values 35 per cent when based on rigid frame action alone. This variation in measured and computed stresses on models tested in the laboratory amounted to as much as 100 per cent.

The measurements obtained in the field investigation of stress distribution in floorbeam hangers on all of the railway bridges tested have made plain that a satisfactory method of analysis must take into account such factors as variation of the section of the member, width of member, influence of brackets or gussets, and the effect of shear distortion on the bending in the member.

Method of analysis developed by the Purdue University staff is a modified slope-deflection solution combined with shear slope in the diaphragm at the

floorbeam connection. With this method of analysis close agreement is secured with static stresses measured in the field and also laboratory models.

Laboratory Tests

A great deal of experimental work has been done in the laboratory on this project. Mention has been already made of the model hanger and floorbeam frames tested to verify the method of analysis.(5) Eight model frames were made of aluminum shapes and were scale models of floorbeams and hangers of actual bridges tested in the field. A frame was also made from a solid plate and tested under several cases of loading. All of these model frame tests showed that the slope of the hanger at the top of the floorbeam was not equal to the slope at the end of the floorbeam, but was much greater—the difference being due to shear slope in the web of the connected hanger length in the joint. So far as is known the fact that shear distortion inside the joint of a frame composed of steel (or metal) members could affect the stresses in the member was here brought out for the first time.

Tests were also made on riveted and bolted lap joints. These tests all showed that very high tensile stress occurs at the side of the holes when rivet bearing without clamping occurs under axial loads. Plates connected by pins bearing in single shear have stresses at the sides of the holes 20 to 40 or more times the average stress on the gross section of the plate, when load is applied centrally in regard to width of plate. In joints connected with high strength bolts the axial stresses in the plate at the outside edge of the washer are equal to about one-fifth those produced at the edge of hole on a similar plate connected by bearing pins in single shear.

Photoelastic tests were also made and the results showed close agreement with stresses secured with SR 4 gages.

These laboratory tests provide a reliable index to the location and magnitude of the stress and strain concentrations that exist at the line of the lower row of rivets connecting the hanger material to the upper gusset plate.

Tests were made in the laboratory of a large full scale structural joint connected with high strength bolts and a joint connected with rivets and high strength bolts. Each joint was composed of two wide-flange beams 12 inch by 40 pounds per foot spliced together by two large gusset plates. The riveted and bolted joint was connected by rivets except that the first two rows of holes at the edges of the gussets corresponding to the two lowest of holes in the hanger connection to the hip gusset, were connected by high strength bolts. The connection of the wide-flange beam to the gussets represents the connection of a full-size floor beam hanger to the upper chord gusset. Each joint thus permitted the testing of two such connections; one to square gussets and one to gussets tapered at the lower ends.

These large joint tests were planned as a study of methods proposed to help prevent fatigue failures in floorbeam hangers of existing bridges specifically, or more generally in single lap structural connections. The results of this investigation indicate that the performance of single lap bolted joints designed in the same way as the bolted joint tested in this investigation is favorable for the reduction of fatigue failure in new construction since—

1) The slip of the end bolts is less than the slip of end rivets. Hence the bolts should only come to bear at higher loads than for riveted joints.

2) At design or service loads major slip should not occur, and hence the bolts should not come to bear.

3) At service loads the bolts may be expected to retain their high clamping, producing a favorable stress and strain distribution in the joint.

For the same reasons the results of this investigation appear favorable for the replacement of the rivets at the ends of the gussets by high strength bolts in an attempt to protect existing structures against fatigue failures. The following precautions are necessary:

1) Avoid damage to the metal around the holes in removing present rivets.
2) Ream out the holes with a sharp reamer so the metal is smooth and unscratched and all damaged material removed.
3) Under no circumstances drift any holes.
4) Make sure that the bolts are tightened into the yield range.
5) Use carburized washers of specified size.

CONCLUSION

The dieselization of railroads, of course, has reduced the loads that older and lighter designed truss bridges will be called on to carry and this has helped to mitigate the floorbeam hanger trouble in the last couple of years.

These tests have shown that the fatigue life of hangers in existing bridges can be extended by increasing the radius of re-entrant cuts at copes. Also by replacing the two lowest rows of rivets in the upper chord gussets with high strength bolts.

Iron and Steel Structures, Com. 15 of A.R.E.A., contemplates a change in the specifications for the design of floorbeam hangers to take into account the actual stresses occurring in these members. It may also be desirable to reduce the allowed unit stress.

Obviously the revised specification will not allow a hanger composed of sections that are only tied together with occasional tie plates.

The research work on this project has been very worth while and will save the railroads several hundred thousand dollars in forestalling failure or extending the fatigue life of floorbeam hangers.

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